JL ENGINEERING, LLC
Civil, Structural, & Consulting

Memo

To: Mr. S. Joseph Darrah, Darrah Law Office, P.C.

From: Justin Lundvall, PE Justin F. Lundvall

Date: May 27, 2021

Re: Pile Driving & Superstructure Issues During Phase I Construction

Project: WY FLAP 6WX(1) -South Fork Shoshone River Bridge, SouthFork Road, Park County, Wyoming

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JL Engineering, LLC has been engaged by Mr. Joseph Darrah, Darrah Law Office, P.C. to provide expert engineering opinions for the disputed Southfork Road Project and in particular to the bridge construction. The bridge construction was part of a bridge replacement project located in Park County, Wyoming over the South Fork of the Shoshone River.

The project consisted of phased construction in removal of an existing bridge structure and the construction of a new structure. The new structure is a 150' (overall) simple span bridge consisting of four welded plate steel girders with total girder length of 149'. The girders were comprised of 3 pieces with a single piece length of 86' and 2 pieces with lengths of 31'-6" each. The pieces were connected together with bolted splice connections. This was part of the superstructure which also included a poured concrete deck. The substructure of the project consisted of reinforced concrete abutments, endwalls, and wingwalls. The concrete abutments were supported by driven steel H-piles.

The design of this project was completed by the Federal Highway Administration (FHWA). The specifications used for the design of this structures were given as the AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014, with interims through 2016. In addition, the construction specifications were the Federal Highway Administration Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP-14, Dual Units. Further information was provided in the way of geotechnical information. This information was found the document the Final Geotechnical Report, South Fork Road WY-FLAP-6WX(1)-18-01 dated December 2018, prepared by the U.S. Department of Transportation, Federal Highway Administration, Central Federal Lands Highway Division, Geotechnical Services Branch.

P.O. Box 3125 • Cody, WY 82414 • 307.272.4399 • justinl5123@hotmail.com

Pile Driving

As far as the actual construction of the Project and document review pile driving is one of the first sources of dispute. The main issue of contention is the claim of differing site conditions leading to the additional length of pile and the necessary equipment required to install the piling to meet the required conditions of the design. The claim was based on a Type 1 differing site condition as described in FAR 52.236-2. FAR 52.236-2 describes a Type 1 condition as:

> "Subsurface or latent physical conditions at the site which differ materially from those indicated in this contract..."

As noted information was available and provided in the way of the aforementioned Geotechnical Report. Two bore holes were conducted for this particular site with one for each abutment. Section 3.3 Summary of Site Conditions, as given in the Report, states that for both abutment locations the subsurface conditions were generally consistent. The Report goes on to describe the conditions as follows:

> "Both borings encountered a layer of fill material consisting of a mixture of clay, sand, gravel and cobbles to depths of about 8 to 12 feet. Underlying the fill material was a layer of low plasticity silty, clayey sand to sandy silt approximately 7 feet thick. Once groundwater was encountered at a depth of 14 to 17 feet, a thick layer of medium dense poorly-graded sand was encountered that extended to a depth of 52 to 53 feet. The poorly-graded sand was predominantly medium to coarse grained, with little gravel or fines. Underlying the poorly-graded sand layer was a silty sand, mostly composed of fine grained sand which eventually graded to a sandy silt to the maximum depths explored, approximately 90 feet."

Boring logs showing the above summarized information were also provided in the Report as well as depicted in the Bridge Plans on page 4 of 30, Boring Logs, Drawing No. RG3141-D.

Additionally, per the Geotechnical Report Section 4.1.7 Pile Drivability states the following:

> "The contractor should conduct drivability analysis using the Wave Equation Analysis Program (WEAP) to select hammers that have sufficient energy to drive the piles to the desired embedment without exceeding the acceptable pile driving stresses and blows per foot presented in the project specifications. Test piles should be driven while instrumented with PDA (Pile Driving Analyzer) to determine production pile driving criteria."

The Geotechnical Report does make estimates as to pile tip elevations and sites these as minimum depths that must be achieved. However, the Report goes on to site Required Nominal Driving Resistance which is a requirement regardless of the estimated minimum depth. The minimum depth elevations and required resistance is also shown in the Bridge Plans on page 3 of 30, Foundation Plan, Drawing No. RG3141-C.

As is generally understood, a geotechnical exploration and report is at best an estimate in trying to determine subsurface conditions based on a limited sample size. This is done in an effort to make the best estimates for foundation recommendations and design. Conditions may vary from the explored conditions in between bore holes and/or along the actual construction area once construction commences. As stated in the Geotechnical Report:

> "The results of these explorations and tests represent conditions at the specific locations indicated. Subsurface conditions between these locations may vary. The Analysis and Recommendations Section in this report include interpretations and recommendations developed by the Government in the process of preparing the design. These interpretations are not intended as a substitute for the personal investigation, independent interpretation, and judgment of the Contractor."

Dynamic pile load testing was an actual separate bid item listed in the estimated quantities. Such testing was specifically performed for Abutment No. 1 between July 31 and August 14 of 2019 and is sited in a letter from SK Geotechnical dated August 21, 2019. In this letter it states that an inspector's chart for control of the remaining production pile was developed. The letter also summarizes the subsurface conditions from the Geotechnical Report/Boring Logs. Of note, there is no mention of differing soil, site conditions, or subsurface conditions as compared to the referenced subsurface conditions during the pile driving test. This testing and inspector's chart development was used to develop the pile driving "plan" for the rest of the structure. Additionally, with no other material testing and no reference to the actual driving being found to be different throughout the pile tests there does not seem to be any information or confirmation that supports the claim that the physical conditions were any different than those indicated in the Geotechnical Report.

The results of the tests as performed indicated that the piles, to obtain the required nominal driving resistance, would have to be longer and that the equipment that FirstMark intended to use and used during testing would not be sufficient for the rest of the driving. Section 551.06 – Pile Lengths, Section 551.07 – Test Piles, and Section 551.08 of the FP-14 Standards and Specifications (FP-14) specifically mention furnishing piles with sufficient length to obtain the required resistance and/or the required nominal pile capacity. Section 551.10 (b) also references furnishing full-length, un-spliced piles for lengths up to 60 feet. There is no mention in any of these sections of absolute pile lengths. The provisions of FP-14 Section 551.18 also discuss how to measure piles by the linear foot as well as splices to drive piles deeper than the estimated tip elevation. Section 551.19 follows up with payment of accepted quantities be paid at the contract price per unit of measurement.

It can be understood that initial planning would be based off of the estimated minimum depths with the provision of full length piles Sections 551.10 (b) (at a minimum for testing to determine the pile driving criteria/plan). Further Section 551.05(b) - Approval of pile-driving equipment, does not mandate what equipment a Contractor may use but what is expected from that equipment. However, with a separate pay item for testing, the references for testing to set up a production pile driving criteria/plan, and the requirement to meet a nominal pile driving resistance requirement and not a pile depth there leaves room for adjustment in pile length and determining if on-site equipment was adequate for required installation per testing results. Additionally, measurement and payment for the piles were set up, as referenced, by linear feet.

Without any other material testing, no noted differences in driving during the testing, and the test results as per the SK Geotechnical Letter it appears as the claim for differing site conditions based on a Type 1 definition is unwarranted. The equipment, at a minimum for the testing, should have been capable of handling full length (60 foot) piles to determine the driving criteria/plan and adjusted from there if necessary.

Girder Deformation

Another item of contention for this Project was the deformation of girders during Phase I construction. Particularly the claim of the "permanent design" not conforming or meeting AASHTO LRFD Design requirements. The claim is based on Section 6.10.3.4.2 – Global Displacement Amplification in Slender I-Girder Bridge Units. This is found in the 2015 Interims to the 2014 LRFD Specifications.

A side note, in Thornton Tomasetti's (TT's) review the specification noted was the 2017 Edition of AASHTO LRFD and not the 2014 Edition (with Interims) that the Bridge was designed under. Additionally, in the Smith Monroe Gray (SMG) Engineers, Inc. calculations that were submitted it appears that the equation used was possibly from the newer 7th edition as well and not the 2015 Interims. It should be noted, that generally, the newer editions incorporate the revisions made in the Interims of the previous editions. However, as observed there is a different factor that was not present in the equation in the 2015 Interims and the limiting percentage was at 50% of the global lateral-torsional resistance rather than 70% presented in the newer edition and used equation. Additional literature review of the paper by Yura et al., 2008 does show the equation and limiting value more in line with the newer edition than the 2015 Interim edition. Overall, it does not appear that the outcome/result as calculated would change just the actual calculated values would be different.

As per the AASHTO Specification Section 6.10.3.4.2, as referenced in the design, terminology is used that the provisions shall apply to spans of I-girder bridge units with three or fewer girders interconnected by cross-frames or diaphragms that also meet additional conditions as listed. As designed the bridge section would meet the criteria as cited in this section for additional checks of the elastic global lateral-torsional buckling resistance of the span acting as a system. This check as mentioned, was completed by SMG and referenced by TT and with the new edition equation triggers a result that would give the designer options. The language used in the 2015 Interim is as follows:

> "Should the sum of the largest total factored positive girder moments across the width of the unit within the span under consideration exceed 50 percent of M_{gs} the following alternatives may be considered:
>
> - The addition of flange level lateral bracing adjacent to supports of the span may be considered as discussed in Article 6.7.5.2;
> - The unit may be revised to increase the system stiffness; or
> - The amplified girder second-order displacements of the span during the deck placement may be evaluated to verify that they are within tolerance permitted by the Owner."

As can be seen in the Specifications use of the word "may" for alternatives to be considered and also for each of the alternatives listed. The word "may" as defined by AASHTO in their Introduction Chapter is as follows:

> "The term "may" indicates a criterion that is usable, but other local and suitably documented, verified, and approved criterion may also be used in a manner consistent with the LRFD approach to bridge design."

It is not certain if this is the approach taken by the Designer/FHWA but it is something to consider. It also may lead to the reliance on Sections of FP-14 particularly Section 555-18 – Erection and Section 562 – Temporary Works.

Subsection 562-03 – Design, states the following:

> "Design temporary works that will support loads imposed and provide the necessary rigidity to produce the lines and grades shown in the plans for the final structure. Design temporary works according to the AASHTO, *Load and Resistance Factor Design (LRFD) Bridge Design Specifications* or AASHTO, *Guide Design Specifications for Bridge Temporary Works*. Ensure the design load on manufactured devices is within the load rating recommended by the manufacturer."

Additionally, Subsection 555.18 – Erection, states to conform to Section 562 for falsework and forms. Further Section 555.18 (c) (3) – Maintaining alignment and camber states the following:

> "Support structural segments to produce the proper alignment and camber in the completed structure. Install cross frames and diagonal bracing during erection to provide stability and ensure correct geometry. **Provide temporary bracing at any stage of erection**." (Emphasis added)

One other location that also reiterates the Contractor's role in bracing and stability of the structure is on the Bridge Plans themselves. This is found on page 2 of 30, General Notes & Estimate, Drawing No. RG3141-B, under the General Notes and Structural Steel Section. The language expressed in the note is as follows:

> "The contractor shall be responsible for the stability of the structure during **all phases of construction**." (Emphasis added)

It is clearly evident in the three locations cited that the Contractor does bear responsibility for the stability of the structure during the construction of this structure.

The argument has been made that if the Engineer of Record or Designer had used lateral bracing in accordance with AASHTO Section 6.10.3.4.2 that temporary bracing would not have been necessary in addition to the permanent cross bracing and the bridge deck formwork.

Various computer modeling results and limited hand calculations have been presented showing that AASHTO Section 6.10.3.4.2 was not met. According to the AASHTO specification this section should be checked with options that "may" be considered. However, the statement that additional temporary bracing would not have been necessary seems to be a broad coverage of the issue. Further, no calculations or model results were given to back this statement.

It is not argued that one of the potential alternatives that is suggested by TT and stated by SMG, that flange lateral bracing is required (although other alternatives "may" be considered), would not have potentially helped the erection and deformation of the girders during the non-composite phase of the construction but to what extent? That absolutely no additional temporary bracing would be required? As stated in the AASHTO commentary for Section 6.10.3.4.2.:

> "The recommendations in this Article are intended to avoid excessive amplification of the lateral and vertical displacements of slender I-girder bridge units during the deck placement operation before the concrete deck has hardened."

This does not rule out any potential need for additional temporary bracing.

In a very simple view of the cross-section of the Stage 1 Construction as depicted on the Bridge Plans page 6 of 30, Stage Construction Abutment, Drawing No. RG3141-F as well as on page 7 of 30, Stage Construction Pier, Drawing No. RG3141-G both show cross sections for the Stage 1 Deck Construction. Looking at the detail provided and the Construction Staging Sequence the cantilever part of the deck has more dead load weight than the right hand side or the area between the Stage 3 Deck Closure Pour and particularly before the temporary concrete barrier is in place which was to be the start of Stage 2 Construction. With this in mind the section would want to potentially move or rotate (lateral displacement and/or deformation) towards the cantilever side due to the unbalanced loading, unless potential additional temporary bracing were applied to keep the section in proper alignment. This bracing could be minimal depending on the actual reaction of the section. This unbalanced loading would be present even with the "suggested/required" permanent lateral bracing option were present.

As was stated the bridge was completed with the plans and specifications as given and bid upon. Additional temporary bracing was used during construction to maintain proper alignment of the girders during the deck pouring. This temporary bracing partially consisted of "off the shelf" rated products including chains and hand operated come alongs to maintain the girder alignment to provide proper rigidity of the system until the deck hardened and further stage construction was completed. As has been demonstrated by the bridge completion, the bridge was constructible to FHWA acceptable standards and tolerances with seemingly minimal temporary bracing and without the "permanent" lateral bracing or other potential alternatives listed. The bridge is currently in service as of this writing.

Encls. AASHTO Section 6.10.3.4.2 – 2015 Interims (3 pages) and 2017 Ed. (2 pages)

References

AASHTO (2012). *AASHTO LRFD Bridge Design Specifications, 6th Edition, with 2013 Interim Revisions*, Washington, D.C.

AASHTO (2014). *2015 Interim Revisions to the AASHTO LRFD Bridge Design Specifications, 7th Edition*, Washington, D.C.

AASHTO (2017). *AASHTO LRFD Bridge Design Specifications, 8th Edition, Article 6.10.3.4.2*. Washington, D.C.

United States Department of Transportation – Federal Highway Administration (2014). *Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP-14*. Washington, D.C.

United States Department of Transportation – Federal Highway Administration (2018). Southfork Road, Park County, WY, WY 6WX(1), Final Geotechnical Report, Report No. WY-FLAP-6WX (1)-18-01

United States Department of Transportation – Federal Highway Administration. Plans for Proposed WY FLAP 6WX(1) South Fork Road, Shoshone National Forest, Park County.

Yura J., Helwig T., Herman R., and Zhou C. (2008). Global Lateral Buckling of I-Shaped Girder Systems. Journal of Structural Engineering, 134:9, 1487-1494.

6.10.3.3—Shear

Revise the following definition in the "where" list of this Article:

V_{cr} = shear-yielding or shear-buckling resistance determined from Eq. 6.10.9.3.3-1 (kip)

6.10.3.4—Deck Placement

Add the following at the beginning of this Article:

6.10.3.4.1—General

C6.10.3.4

Change this Article number to "*C6.10.3.4.1*"

Change Eqs. C6.10.3.4-1, C6.10.3.4-2 and C6.10.3.4-3 to Eqs. C6.10.3.4.1-1, C6.10.3.4.1-2 and C6.10.3.4.1-3 in this Article.

In paragraph 2 of this Article, change Eq. C6.10.3.4-1 to Eq. C6.10.3.4.1-1 (at two locations).

In this Article in the paragraph underneath Eq. C6.10.3.4-3, change Eq. C6.10.3.4-2 and Eq. C6.10.3.4-3 to Eq. C6.10.3.4.1-2 and Eq. C6.10.3.4.1-3.

Add the following article to this Section:

6.10.3.4.2—Global Displacement Amplification in Slender I-Girder Bridge Units

The provisions of this Article shall apply to spans of I-girder bridge units with three or fewer girders, interconnected by cross-frames or diaphragms, that also meet both of the following conditions in their noncomposite condition during the deck placement operation:

- The unit is not braced by other structural units and/or by external bracing within the span; and
- The unit does not contain any flange level lateral bracing or lateral bracing from a hardened composite deck within the span.

Considering all of the girders across the width of the unit within the span under consideration, the sum of the largest total factored positive girder moments during the deck placement should not exceed 50 percent of the elastic global lateral-torsional buckling resistance of the span acting as a system. The elastic global lateral-torsional buckling resistance of the span acting as a system, M_{gs}, may be calculated as follows:

$$M_{gs} = \frac{\pi^2 w_g E}{L^2}\sqrt{I_{eff} I_x} \quad (6.10.3.4.2\text{-}1)$$

in which:

- For doubly symmetric girders:

$$I_{eff} = I_y \quad (6.10.3.4.2\text{-}2)$$

- For singly symmetric girders:

$$I_{eff} = I_{yc} + \left(\frac{t}{c}\right) I_{yt} \quad (6.10.3.4.2\text{-}3)$$

where:

c = distance from the centroid of the noncomposite steel section under consideration to the centroid of the compression flange (in.). The distance shall be taken as positive.

I_x = noncomposite moment of inertia about the horizontal centroidal axis of a single girder within the span under consideration (in.^{4})

I_{yc}, I_{yt} = moments of inertia of the compression and tension flange, respectively, about the vertical centroidal axis of a single girder within the span under consideration (in.^{4})

I_y = noncomposite moment of inertia about the vertical centroidal axis of a single girder within the span under consideration (in.^{4})

L = length of the span under consideration (in.)

t = distance from the centroid of the noncomposite steel section under consideration to the centroid of the tension flange (in.). The distance shall be taken as positive.

w_g = girder spacing for a two-girder system or the distance between the two exterior girders of the unit for a three-girder system (in.)

Should the sum of the largest total factored positive girder moments across the width of the unit within the span under consideration exceed 50 percent of M_{gs}, the following alternatives may be considered:

- The addition of flange level lateral bracing adjacent to the supports of the span may be considered as discussed in Article 6.7.5.2;
- The unit may be revised to increase the system stiffness; or
- The amplified girder second-order displacements of the span during the deck placement may be evaluated to verify that they are within tolerances permitted by the Owner.

Add the following article to this Section:

C6.10.3.4.2

The recommendations in this Article are intended to avoid excessive amplification of the lateral and vertical displacements of slender I-girder bridge units during the deck placement operation before the concrete deck has hardened. The elastic global buckling resistance may be used as an indicator of the susceptibility of general straight, curved and/or skewed I-girder systems to second-order amplification under noncomposite loading conditions (White et al., 2012). The global buckling mode in this case refers to buckling of the bridge unit as a structural unit, and not buckling of the girders between intermediate braces. Limiting the sum of the total factored positive girder moments across the width of the unit within the span under consideration to 50 percent of the elastic global buckling resistance of the span acting as a system theoretically limits the amplification under the corresponding nominal loads to a maximum value of approximately 1.5.

Eq. 6.10.3.4.2-1 (Yura et al., 2008) provides one method of estimating the elastic global lateral-torsional buckling resistance of a given I-girder bridge span under noncomposite loading conditions. Two-girder units are particularly susceptible to excessive global lateral-torsional amplification during the deck placement; however, units with large span/width ratios having up to three girders also may be susceptible to significant global amplification in some cases. Other methods, such as an eigenvalue buckling analysis or a global second-order load-deflection analysis, may also be used to determine the response of the system. Once a concrete deck is acting compositely with the steel girders, a given span of a bridge unit is practically always stable as an overall system; Eq. 6.10.3.4.2-1 is not intended for application to I-girder bridge spans in their composite condition. Eq. 6.10.3.4.2-1 is also not applicable to I-girder bridge units with more than three girders, which are typically not susceptible to excessive global lateral-torsional amplification during the deck placement.

Eq. 6.10.3.4.2-1 was derived assuming prismatic girders and that all girder cross-sections in the unit are the same. For cases where the girders are nonprismatic and/or the girder cross-sections vary across the unit, it is recommended herein that length-weighted average moments of inertia within the positive-moment sections of all the girders in the span under consideration be used for I_x, I_y, I_{yc} and I_{yt}, as applicable, in calculating the elastic global lateral-torsional buckling resistance from Eq. 6.10.3.4.2-1. Also, in cases where the girder spacing is less than the girder depth, it is recommended that the more general elastic global lateral-torsional buckling equation provided in Yura et. al. (2008) be used, as Eq. 6.10.3.4.2-1 becomes more conservative in this case. Yura et al. (2008) further indicates the adjustments that need to be made to the more general buckling equation for singly symmetric girders and/or for three-girder systems.

Large global torsional rotations signified by large differential vertical deflections between the girders and also large lateral deflections, as determined from a first-order analysis, are indicative of the potential for significant second-order global amplification. Situations exhibiting potentially significant global second-order amplification include phased construction involving narrow unsupported units with only two or three girders and possibly unevenly applied deck weight. One suggested method of increasing the global buckling resistance in such cases is to consider the addition of flange level lateral bracing to the system. Yura et al. (2008) suggest adjustments to be made when estimating the elastic

global lateral-torsional buckling resistance of the system where a partial top-flange lateral bracing system is present at the ends of the span, along with some associated bracing design recommendations.

6.10.3.4.2—Global Displacement Amplification in Narrow I-Girder Bridge Units

The provisions of this Article shall apply to spans of straight I-girder bridge units with three or fewer girders, interconnected by cross-frames or diaphragms, that also meet both of the following conditions in their noncomposite condition during the deck placement operation:

- the unit is not braced by other structural units and/or by external bracing within the span; and
- the unit does not contain any flange level lateral bracing or lateral bracing from a hardened composite deck within the span.

Considering all of the girders across the width of the unit within the span under consideration, the sum of the largest total factored girder moments during the deck placement within the span under consideration should not exceed 70 percent of the elastic global lateral-torsional buckling resistance of the span acting as a system. The elastic global lateral-torsional buckling resistance of the span acting as a system, M_{gs}, may be calculated as follows:

$$M_{gs} = C_{bs} \frac{\pi^2 w_g E}{L^2} \sqrt{I_{eff} I_x} \qquad (6.10.3.4.2\text{-}1)$$

in which:

C_{bs} = system moment gradient modifier
= 1.1 for simply-supported units
= 2.0 for continuous-span units

- For doubly symmetric girders:

$$I_{eff} = I_y \qquad (6.10.3.4.2\text{-}2)$$

- For singly symmetric girders:

$$I_{eff} = I_{yc} + \left(\frac{t}{c}\right) I_{yt} \qquad (6.10.3.4.2\text{-}3)$$

where:

c = distance from the centroid of the noncomposite steel section under consideration to the centroid of the compression flange (in.). The distance shall be taken as positive.

I_x = noncomposite moment of inertia about the horizontal centroidal axis of a single girder within the span under consideration (in.^{4})

I_{yc}, I_{yt} = moments of inertia of the compression and tension flange, respectively, about the vertical centroidal axis of a single girder within the span under consideration (in.^{4})

C6.10.3.4.2

The recommendations in this Article are intended to avoid excessive amplification of the lateral and vertical displacements of narrow straight I-girder bridge units during the deck placement operation before the concrete deck has hardened. The global buckling mode in this case refers to buckling of the bridge unit as a structural unit, and not buckling of the girders between intermediate braces. Limiting the sum of the largest total factored girder moments across the width of the unit within the span under consideration to 70 percent of the elastic global buckling resistance of the span acting as a system theoretically limits the amplification under the corresponding nominal loads to a maximum value of approximately 2.0.

Eq. 6.10.3.4.2-1 (Yura et al., 2008) provides one method of estimating the elastic global lateral-torsional buckling resistance of a given straight I-girder bridge span under noncomposite loading conditions. The system moment gradient modifier, C_{bs}, in Eq. 6.10.3.4.2-1 accounts for the beneficial effect of the moment gradient within the span on the elastic global lateral-torsional buckling resistance of the span acting as a system, which is particularly significant for continuous-span units. A C_{bs} value of 1.1 applies to simply-supported units, and should also be applied if investigating continuous-span units that are in a partially erected condition. A C_{bs} value of 2.0 applies to fully erected continuous-span units. Two-girder units are particularly susceptible to excessive global lateral-torsional amplification during the deck placement; however, units with large span/width ratios having up to three girders also may be susceptible to significant global amplification in some cases. Other methods, such as an eigenvalue buckling analysis or a global second-order load-deflection analysis, may also be used to determine the response of the system. Once a concrete deck is acting compositely with the steel girders, a given span of a bridge unit is practically always stable as an overall system; Eq. 6.10.3.4.2-1 is not intended for application to I-girder bridge spans in their composite condition. Eq. 6.10.3.4.2-1 is also not applicable to I-girder bridge units with more than three girders, which are typically not susceptible to excessive global lateral-torsional amplification during the deck placement.

Eq. 6.10.3.4.2-1 was derived assuming prismatic girders and that all girder cross-sections in the unit are the same. For cases where the girders are nonprismatic and/or the girder cross-sections vary across the unit, it is recommended herein that length-weighted average moments of inertia within the positive-moment sections of all the girders in the span under consideration be used for I_x, I_y, I_{yc} and I_{yt}, as applicable, in calculating the elastic global lateral-torsional buckling resistance from Eq. 6.10.3.4.2-1. Also, in cases where the girder spacing is less than the girder depth, it is recommended that the more general elastic global lateral-torsional buckling equation

I_y = noncomposite moment of inertia about the vertical centroidal axis of a single girder within the span under consideration (in.^{4})

L = length of the span under consideration (in.)

t = distance from the centroid of the noncomposite steel section under consideration to the centroid of the tension flange (in.). The distance shall be taken as positive.

w_g = girder spacing for a two-girder system or the distance between the two exterior girders of the unit for a three-girder system (in.)

Should the sum of the largest total factored girder moments across the width of the unit within the span under consideration exceed 70 percent of M_{gs}, the following alternatives may be considered:

- The addition of flange level lateral bracing adjacent to the supports of the span may be considered as discussed in Article 6.7.5.2;
- The unit may be revised to increase the system stiffness; or
- The amplified girder second-order displacements of the span during the deck placement may be evaluated to verify that they are within tolerances permitted by the Owner.

provided in Yura et. al. (2008) be used, as Eq. 6.10.3.4.2-1 becomes more conservative in this case. Yura et al. (2008) further indicates the adjustments that need to be made to the more general buckling equation for singly symmetric girders and/or for three-girder systems.

Large global torsional rotations signified by large differential vertical deflections between the girders and also large lateral deflections, as determined from a first-order analysis, are indicative of the potential for significant second-order global amplification. Situations exhibiting potentially significant global second-order amplification include phased construction involving narrow unsupported units with only two or three girders and possibly unevenly applied deck weight. One suggested method of increasing the global buckling resistance in such cases is to consider the addition of flange level lateral bracing to the system. Yura et al. (2008) suggest adjustments to be made when estimating the elastic global lateral-torsional buckling resistance of the system where a partial top-flange lateral bracing system is present at the ends of the span, along with some associated bracing design recommendations.

The elastic global buckling resistance should only be used as a general indicator of the susceptibility of horizontally-curved I-girder systems to second-order amplification under noncomposite loading conditions. Narrow horizontally-curved I-girder bridge units that meet both of the conditions stated in this article in their noncomposite condition during the deck placement may be subject to significant second-order amplification and should instead be analyzed using a global second-order load-deflection analysis to evaluate the behavior. As an alternative, the addition of flange level lateral bracing adjacent to the supports of the span may be considered as discussed in Article 6.7.5.2, or the unit can be braced to other structural units or by external bracing within the span.

6.10.3.5—Dead Load Deflections

The provisions of Article 6.7.2 shall apply, as applicable.

C6.10.3.5

If staged construction is specified, the sequence of load application should be recognized in determining the camber and stresses.

6.10.4—Service Limit State

6.10.4.1—Elastic Deformations

The provisions of Article 2.5.2.6 shall apply, as applicable.

C6.10.4.1

The provisions of Article 2.5.2.6 contain optional live load deflection criteria and criteria for span-to-depth ratios. In the absence of depth restrictions, the span-to-depth ratios should be used to establish a reasonable minimum web depth for the design.